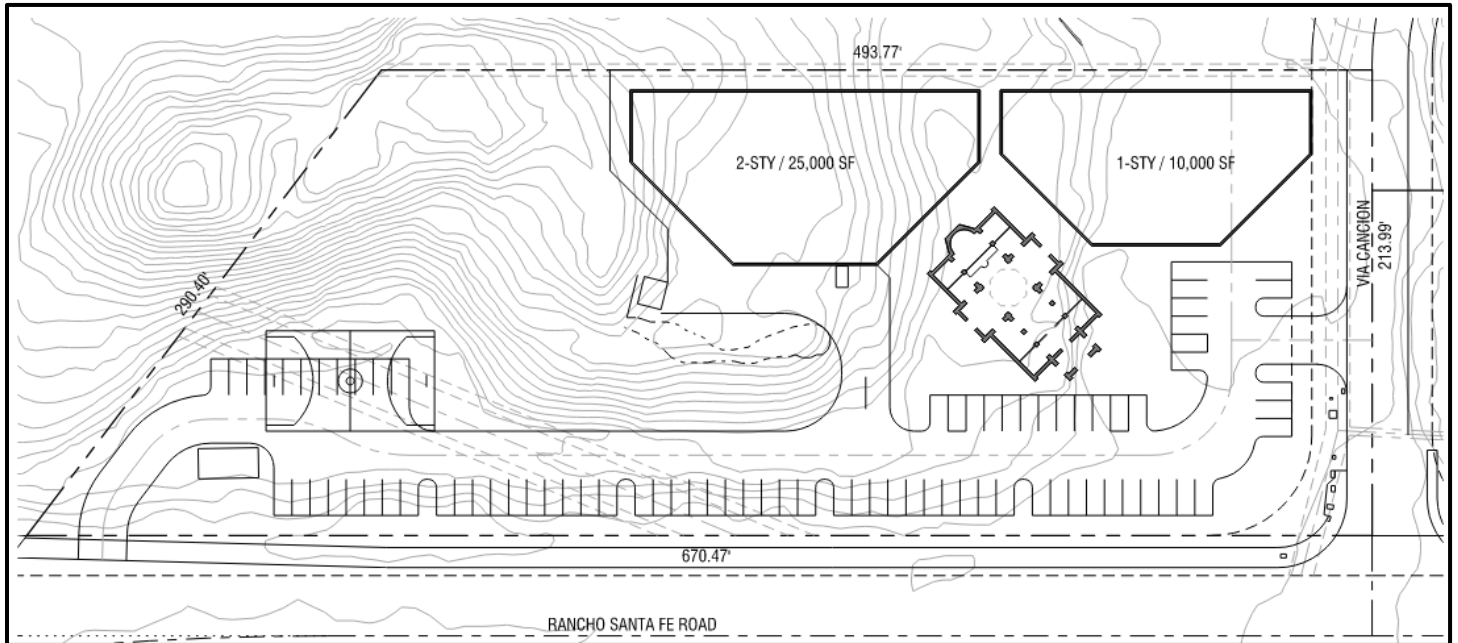


DUE DILIGENCE GEOTECHNICAL ASSESSMENT

St. Katherine of Alexandria Orthodox Church
1690 Via Cancion, San Marcos, California 92078



Submitted to:
Ms. Mehrnoush Yavary, PE
Howes | Weiler | Landy Planning & Engineering
2888 Loker Avenue East, Suite 217
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NOVA Project No. 2023084
July 21, 2023



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Ms. Mehrnoush Yavary, PE
Howes | Weiler | Landy Planning & Engineering
2888 Loker Avenue East, Suite 217
Carlsbad, California 92010

July 21, 2023
NOVA Project No. 2023084

Subject: Due Diligence Geotechnical Assessment
St. Katherine of Alexandria Orthodox Church
1690 Via Cancion
San Marcos, California 92078

Dear Ms. Yavary:

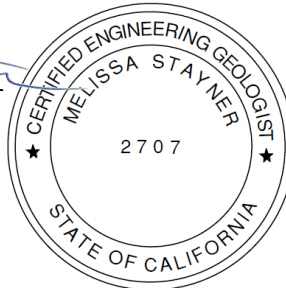
NOVA Services, Inc. (NOVA) is pleased to present herewith the above-referenced report. The report was completed by NOVA for Howes | Wheeler | Landy Planning & Engineering in general conformance with the scope of work presented in NOVA's January 30, 2023, as authorized on May 2, 2023.

NOVA appreciates the opportunity to be of service to HWL and St. Katherine's on this project. If you have any questions regarding this report, please contact us at 858.292.7575 x 413.

Sincerely,

NOVA Services, Inc.

Melissa Stayner, PG, CEG
Senior Engineering Geologist



John F. O'Brien, PE, GE
Principal Geotechnical Engineer



Danielle Brown
Staff Geologist



DUE DILIGENCE GEOTECHNICAL ASSESSMENT

St. Katherine of Alexandria Orthodox Church
1690 Via Cancion, San Marcos, CA 92078

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1. INTRODUCTION

1.1. Terms of Reference

This report presents the findings of a geotechnical assessment by NOVA Services, Inc. (NOVA) of the suitability of a ± 3.13 -acre property located at 1690 Via Cancion, in the City of San Marcos, California (hereinafter 'the site' or 'the property'). St. Katherine of Alexandria Orthodox Church is contemplating purchase of the subject property for development of a new church and related infrastructure.

This report was completed by NOVA for Howes | Wheeler | Landy Planning & Engineering ('HWL') in general conformance with the scope of work presented in NOVA's January 30, 2023 proposal, as authorized on May 2, 2023. When building locations and grading plans are finalized, NOVA should provide an updated report specific to the proposed development.

Figure 1-1 depicts the site vicinity. Figure 1-2 on the following page shows the site location on a recent aerial image.



Figure 1-1. Site Vicinity

1.2. Objective and Limitations

1.2.1 Objective

The objective of NOVA's work is to assess the suitability of the site for the planned church, identifying geotechnical-related requirements for requirements for development.



Figure 1-2. Site Location

1.2.2 Limitations

This report is limited only to a geotechnical-related assessment of the subject property, undertaken for its stated, limited objective only. Future work to develop or otherwise modify the property will require more in-depth geotechnical investigation.

The report does not address environmental considerations, including assessment of the presence of hazardous or toxic materials in the soil, soil gas, groundwater, or surface water within or beyond the site. Such assessment is normally undertaken in separate due diligence after ASTM E1527

2. SCOPE OF WORK

2.1. Field Investigation

2.1.1 Geologic Reconnaissance

Prior to conducting the geotechnical borings, a NOVA geologist completed a reconnaissance of the property and its vicinity, observing the area for evident geologic, groundwater or surface water anomalies.

2.1.2 Subsurface Exploration

The subsurface exploration consisted of drilling seven geotechnical borings (B-1 through B-7) to depths up to 27 feet below the ground surface (bgs) using a truck-mounted drill rig equipped with a hollow-stem auger. Borings B-6 and B-7 did not advance past 2 feet bgs due to drill refusal on large boulders within the rock fill located the southern portion of the site. Boring relocation of B-6 was not possible due to the easement for a pressurized sewer line and obstacles pertaining to the operation of the existing plant nursery. Boring B-7 was re-attempted multiple times, with refusal encountered each time. Figure 2-1 presents the approximate locations of the borings and the alignment of the sewer easement.

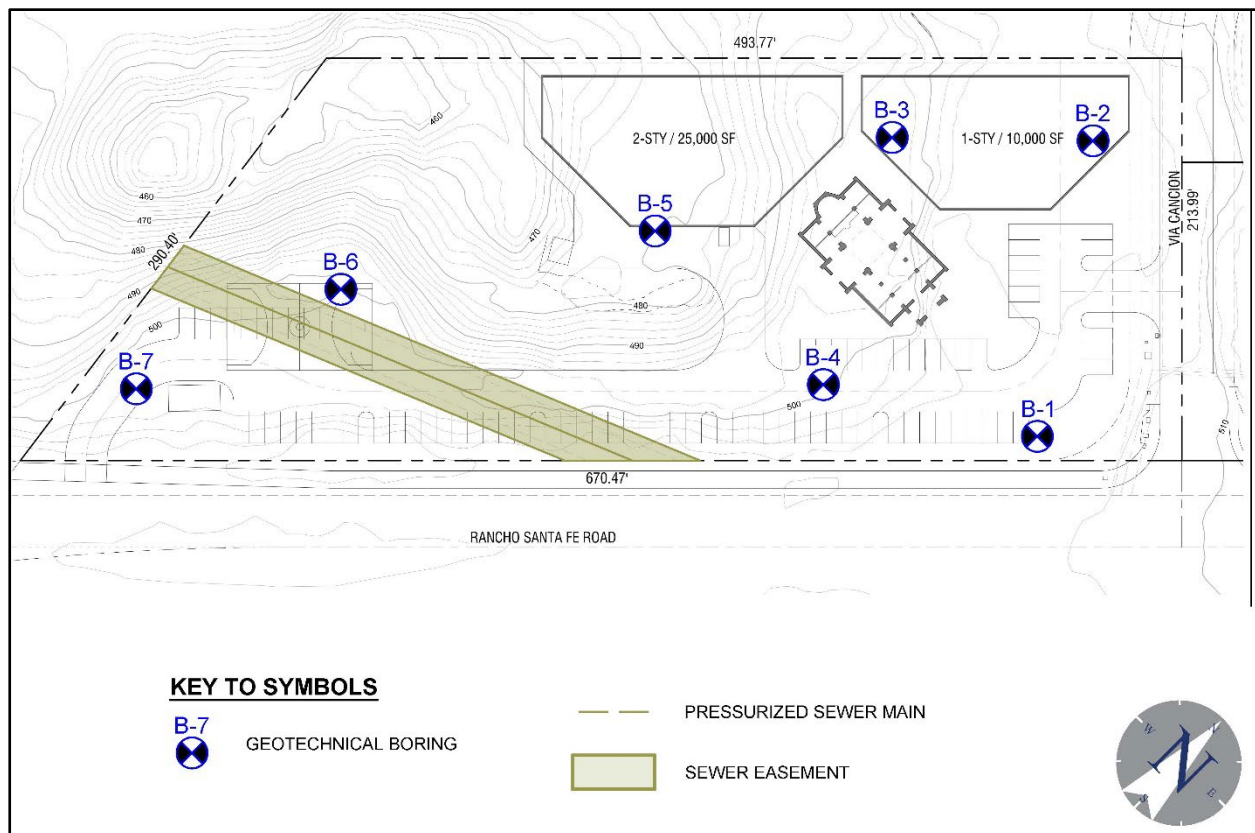


Figure 2-1. Subsurface Exploration Map

The NOVA geologist logged the borings and collected samples of the materials encountered for laboratory testing. Disturbed sampling and *in situ* testing were conducted as abstracted below.

1. Disturbed sampling and *in situ* testing were conducted using the Standard Penetration Test (SPT, after ASTM D1586). The SPT drives a 2-inch outer diameter and 1 $\frac{3}{8}$ -inch inner diameter split tube sampler. The SPT sampler was driven using an automatic hammer with a calibrated Energy Transfer Ratio (ETR) of 73.9%. The number of blows needed to drive the sampler 18 inches is noted in three, 6-inch intervals on the logs. Sampler refusal was encountered when 50 blows were applied during any one of the three 6-inch intervals, a total of 100 blows was applied, or there was no discernible sampler advancement during the application of ten successive blows. The field blow counts, *N*, were corrected to a standard hammer (cathead and rope) with a 60% ETR. The corrected blow counts are noted on the boring logs as *N*₆₀.
2. Bulk samples were obtained from the drill cuttings.

Logs of the borings are presented in Appendix B. Soils are classified according to the Unified Soil Classification System, after ASTM D2487.

2.2. Laboratory Testing

NOVA tested selected samples to evaluate soil classification, engineering properties and develop geotechnical conclusions and recommendations. The laboratory tests consisted of particle-size distribution, Atterberg limits, expansion indexes, and corrosivity.

The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix C.

2.3. Analysis and Report Preparation

The results of the field and laboratory testing were evaluated to develop NOVA's assessment of the site, addressing the geotechnical aspects of the prospective construction. This report presents a record of all work and geotechnical-related recommendations for the prospective construction.

3. SITE AND PROJECT DESCRIPTION

3.1. Site Description

3.1.1 Location and Current Use

The site is a ± 3.13 -acre lot (HWL, 2023) located at 1690 Via Cancion, in the City of San Marcos, California. The site is surrounded by mixed residential developments and roadways. It is bounded by residential homes to the north, Via Cancion to the northeast, South Rancho Santa Fe Road to the south, and Brighton Glen Road to the southwest. A sewer line easement crosses the southwestern portion of the site.

The site is currently occupied by various structures such as a mobile home, storage buildings, a cellular tower, and concrete ponds/waterfalls. The site is currently being used as a nursery and equipment yard for a landscaping business. Figure 3-1 through Figure 3-3 present views of the site conditions existing in May 2023.



Figure 3-1. Existing, Middle of Site, May 2023



Figure 3-2. Existing, SW End of Site, May 2023

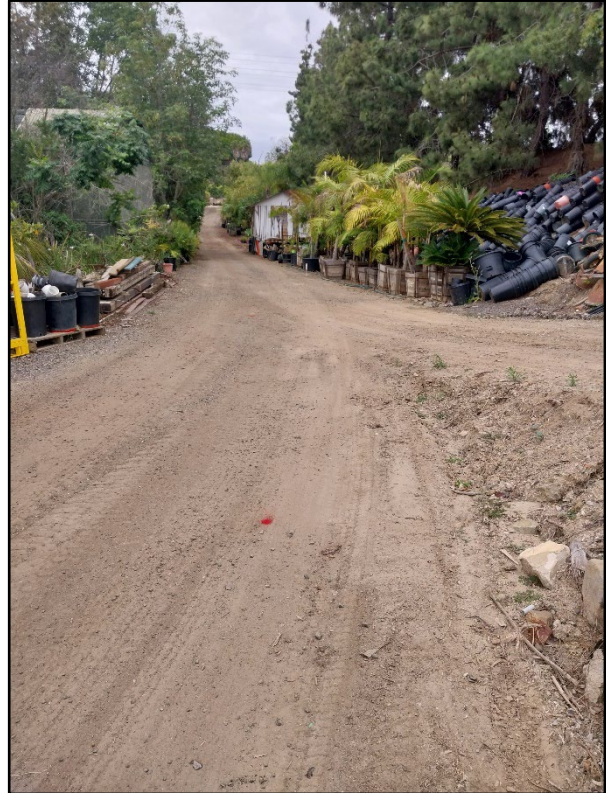


Figure 3-3. Existing, Access Road, May 2023

3.1.2 Historic Use

Review of historic aerial photography indicates clearing efforts occurred in 1985, with fill having been placed by the 1986 photo. Photos from 1994 show agriculture and nursery structures across the entirety of the site, which reflects the current configuration.

3.1.3 Topography

The northern portion of the site has three terraced pads that are relatively flat, which descend to the south. The northern pad is at approximately 504 feet mean sea level (msl), the middle pad is at approximately 498 feet msl, and the southern pad is at approximately 480 feet msl. Along the eastern margin of the site is an access road that becomes narrow in the center and widens out at the southern portion of the site. This road is around 500 feet msl in the southern portion of the site. West of this access road and south of the lowest pad, the topography descends into the pre-existing canyon. The lowest point of the site is at an elevation of approximately 460 feet msl. Figure 3-4 depicts the topography described above.

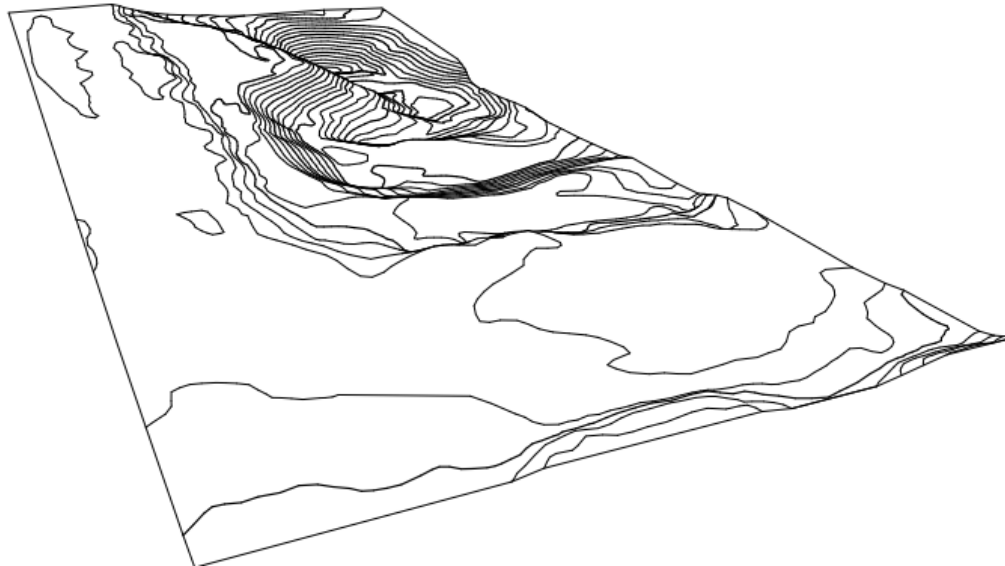


Figure 3-4. Site Topography, View to the Southwest
(source: CTA 2023, Figure A1.1)

3.2. Proposed Development

3.2.1 Architectural

NOVA's understanding of current planning is based upon review of a preliminary architectural schematic drawings provided by CTA 2023. Figure 3-5 reproduces that graphic.

As may be seen by Figure 3-5, the church may be developed adjacent to two proposed at-grade buildings. Pavements for drive aisles and parking stalls will also be constructed.

No below-grade building construction is currently planned. The 6,000 square foot (sf) church will be a single-level structure. The building southwest of the church will be two-stories, approximately 25,000 sf in area. The building northwest of the church will be one-story, approximately 10,000 sf in area. Additional site improvements are anticipated to include driveways, parking lots, pedestrian hardscape, and underground utilities.

3.2.2 Structural

No structural information is available at this early stage of project development. Column loads at the church will vary with column spacing. It is expected that these loads will be in the range 100 kips to 300 kips for exterior and interior columns, respectively. Wall loads may be in the range 2 kips to 5 kips per lineal foot, depending upon the height of the structure and the materials of wall construction.

Column loads to the two buildings should also be relatively light. It is expected that exterior columns will carry loads in the range 50 to 75 kips, while interior columns carry loads about twice that value. Wall loads may be in the range 1 kips to 2 kips per lineal foot.

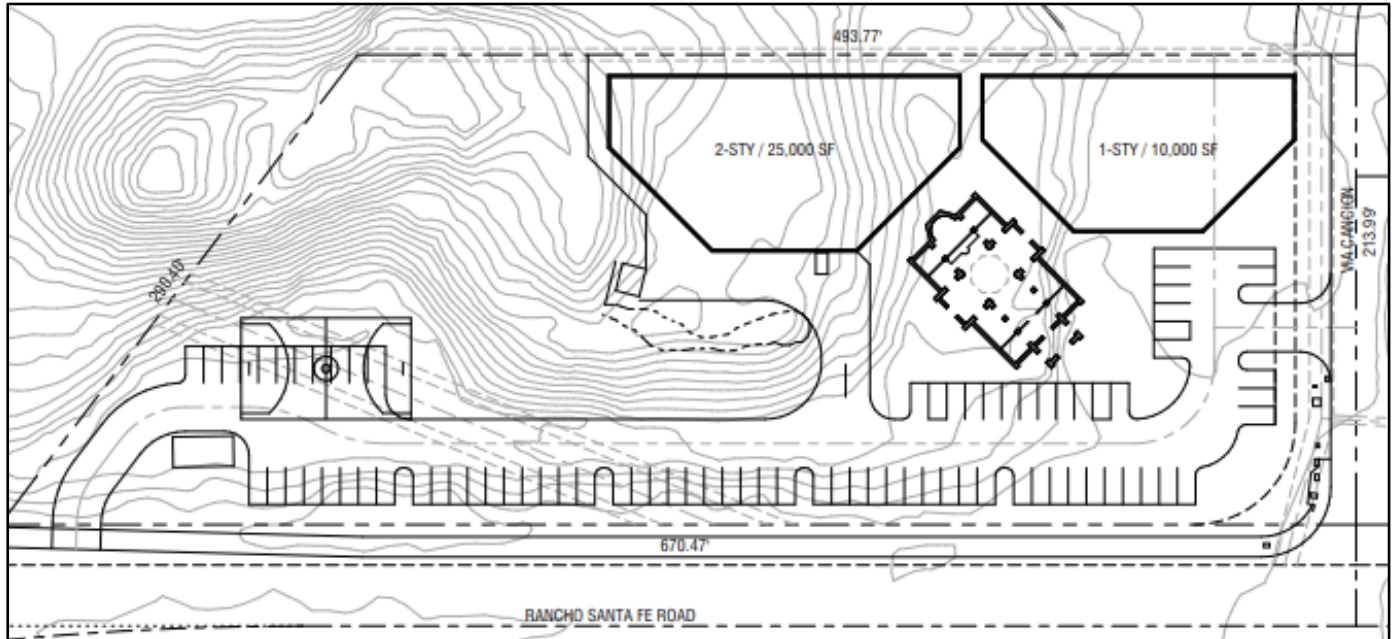


Figure 3-5. Proposed Development
 (source: CTA 2023, Figure A1.0)

3.2.3 Earthwork

No finish grades are available for NOVA's review at this time; however, we anticipate that site grades will be designed to conform to existing grades to the degree practical. Fairly significant earthwork and grading operations will be required to achieve design grades for the structures. It appears the maximum cuts and fills will be in the area of the two-story building, which may require 13 feet of cut on the northern side, and 11 feet of fill on the south side in order to achieve the grade of the existing pad (480 feet msl). Subgrade preparation prior to vehicular pavements and/or pedestrian hardscape construction will also be required.

3.2.4 Stormwater

No planning for stormwater management is available at this time.

4. GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geology

The site is located within the coastal plain of the Peninsular Ranges Geomorphic Province of California, which stretches from the Los Angeles basin to the tip of Baja California in Mexico. This province is characterized as a series of northwest-trending mountain ranges separated by subparallel fault zones and a coastal plain of subdued landforms. The coastal plain is underlain by subsequently deposited marine and nonmarine sedimentary formations. Accelerated fluvial erosion during periods of heavy rainfall, along with the lowering of base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons that characterize the landforms in western San Diego County. The central and eastern portion of the county are underlain by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the Peninsular Ranges Batholith.

Regional geologic mapping indicates the site is underlain by Mesozoic metasedimentary and metavolcanic rocks, undivided (Mzu), and possibly mid-Cretaceous leucogranodiorite of Lake Hodges (Klh) along the southern portion of the site, though this unit was not encountered in our borings. For the purpose of this report, NOVA assumes the whole site is underlain by Mzu. Figure 4-1 presents the regional geology in the vicinity of the site.

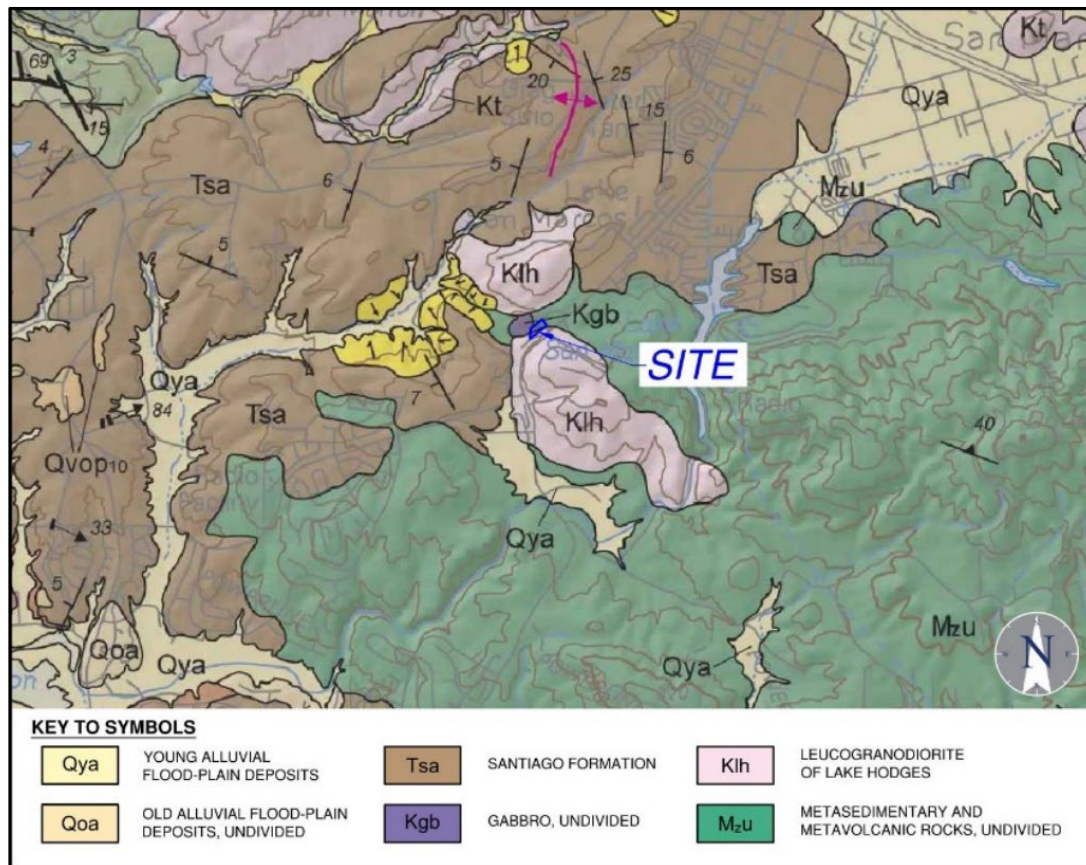


Figure 4-1. Regional Geology Map

4.2. Site-Specific Subsurface

4.2.1 Soil and Rock Units

As encountered in the borings, the site is underlain by fill and metasedimentary/metavolcanic rock. Descriptions of the materials encountered are presented below. Plate 1 presents a geotechnical map. Plate 2 estimates the depth of fill across the site based on a County of San Diego topographic map from 1975. Depth of fill was determined by subtracting the 1975 contours from existing elevations on Google Earth.

Undocumented Fill (af): Fill was encountered in each of the borings to depths between about 12 and 26½ feet bgs. As encountered in the borings, the fill is heterogeneous, containing zones of loose to medium dense clayey fine sand, medium dense to dense poorly graded gravel, and soft to very stiff sandy lean clay and sandy fat clay. Concrete fragments and roots were encountered in some samples. NOVA has no records regarding the placement and compaction of the fill; therefore, it is considered 'undocumented' and at risk for the wide variations in quality and consistency that were observed in the samples from the borings.

Borings B-6 and B-7 encountered refusal on large rock in the fill, leading us to conclude that the southern portion of the site is underlain by an undocumented rock fill, containing oversized material.

This fill encountered is not considered suitable in its existing condition for support of the proposed structures. Figure 4-2 and Figure 4-3 depict the fill encountered in Borings B-1 and B-2.



Figure 4-2. Fill in Boring B-1



Figure 4-3. Fill in Boring B-2

Mesozoic Metasedimentary and Metavolcanic Rock, Undivided (Mzu): The metasedimentary and metavolcanic rock was encountered beneath the fill in Borings B-1 through B-5, to the maximum-explored depth up to about 27 feet bgs. As encountered in the borings, the rock generally consisted of dense to very dense silty and clayey sandstone and hard sandy siltstone. Figure 4-4 and Figure 4-5 are photographs of the formational rock encountered in Borings B-3 and B-4.



Figure 4-4. Formational Rock in Boring B-3



Figure 4-5. Formational Rock in Boring B-4

4.2.2 Groundwater

Groundwater was encountered in Borings B-2 and B-4 at depths of about 12½ and 16 feet bgs, respectively, corresponding to elevations of about 492 and 484 feet msl. Both borings were drilled in areas where pre-existing drainages were identified on a San Diego County historical topographic survey from 1975 (SDCC, 1975). Plate 2, following the text of this report presents the approximate locations of the borings as they relate to the pre-existing drainages. Groundwater likely follows the fill/bedrock contact in the center of the old drainage below the site.

Groundwater and/or groundwater seepage conditions may vary in the future due to changes in land use and/or following periods of heavy rain. This water is anticipated to be encountered during construction of deep foundations. Other perched water conditions not identified during this investigation may be encountered during grading.

Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local landscape irrigation or precipitation, especially during rainy seasons. Groundwater seepage is difficult to predict.

5. REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

5.1. Overview

This section provides a review of geologic, soil and siting-related hazards common to this region of California, considering each for its potential to affect development of the church. Two hazards were identified by this review, as abstracted below.

1. Strong Ground Motion. The site is at risk for moderate-to-severe ground shaking in response to a large-magnitude earthquake ($M \sim 7$) during the lifetime of the planned structures. While strong ground motion could affect the site, there is no risk of liquefaction or related seismic phenomena. The expectation of strong ground motion is common to all civil works in this area of California.
2. Compressible Soils. Relatively thicker (to perhaps 35 feet thickness) fills of clayey soils have been placed across the site. No records of the fill placement are available, such that this fill is considered 'undocumented', subject to wide variations in quality and consistency. In particular, the softer clayey portions of this fill that occur in the near subsurface will be compressible under loads from any new fills or loads from the planned structures.

The following subsections describe NOVA's review of soil and geologic hazards.

5.2. Geologic Hazards

5.2.1 Strong Ground Motion

The tectonic setting of the metropolitan San Diego area includes major north and northwest striking fault zones, the most prominent and active of which is the Newport-Inglewood-Rose Canyon (NIRC) Fault Zone. The NIRC Fault Zone can generate earthquakes of magnitude $M_w = 6.99$.

A web-based analytical tool provided by the American Society of Civil Engineers (ASCE) indicates the site may be subjected to a risk-based Peak Ground Acceleration (PGA_M) of $PGA_M \sim 0.5 g$.

5.2.2 Faulting and Surface Rupture

The site is not located within a designated Alquist-Priolo Earthquake Zone. No known active faults are mapped below the site area, nor did geologic reconnaissance reveal any surface evidence of faulting. The nearest active fault is the Oceanside section of the NIRC, located about 8.7 miles to the southwest.

Due to the lack of active faulting, the probability of fault rupture at this site is considered very low. Figure 5-1 reproduces mapping of regional faulting in the site vicinity.

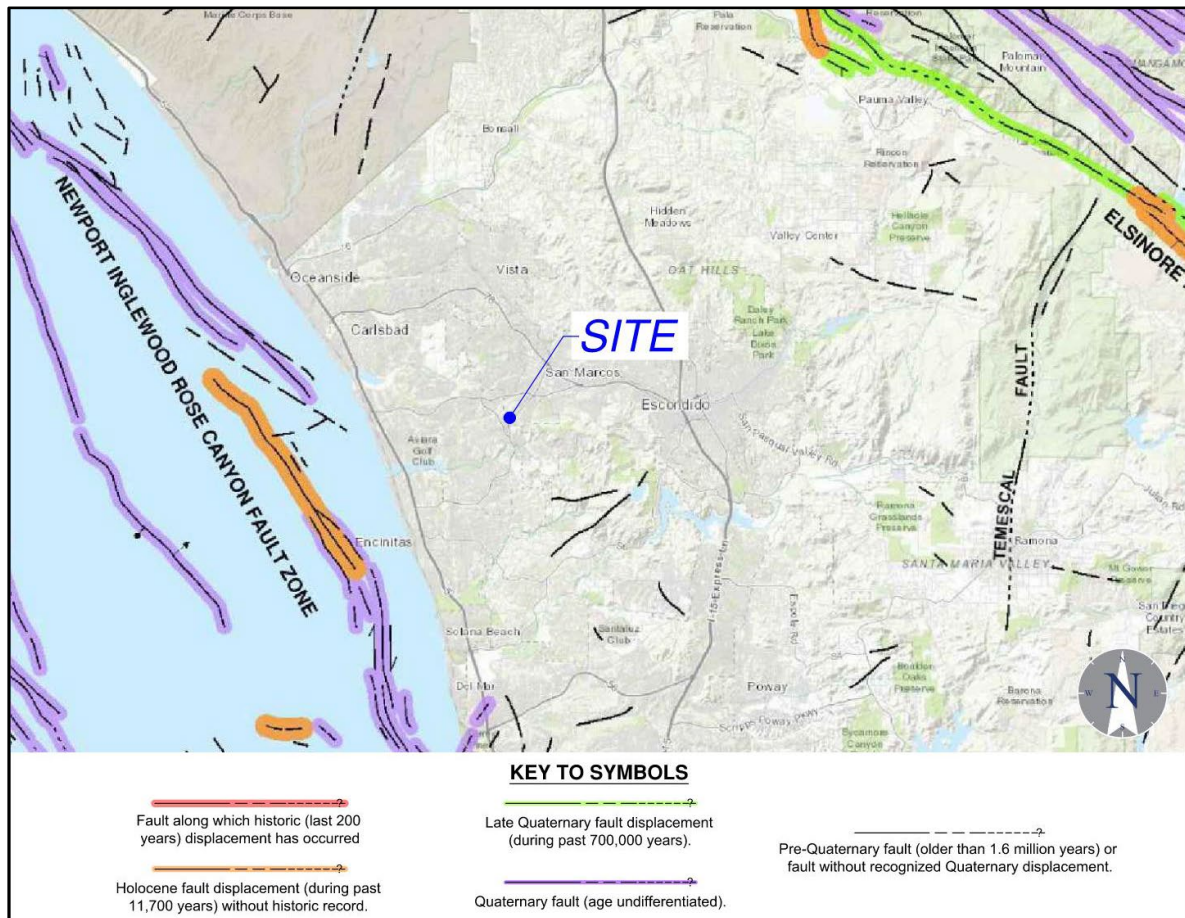


Figure 5-1. Regional Faulting in the Site Vicinity
 (Source: Fault Activity Map of California – California Geological Survey)

5.2.3 Landslides

Evidence of landslides, deep-seated landslides, or slope instabilities were not observed at the time of the field investigation. Additionally, there are no mapped landslides in the vicinity of the project site. The potential for landslides or slope instabilities to occur at the site is considered very low.

5.2. Soil Hazards

5.2.4 Embankment Stability

As used herein, 'embankment stability' is intended to mean the safety of localized natural or man-made embankments against failure. Unlike landslides described above, embankment stability can include smaller scale slope failures such as erosion-related washouts and more subtle, less evident processes such as soil 'creep'.

Geologic reconnaissance of the site showed no indications of localized slope instabilities or slope creep.

5.2.5 Liquefaction and Seismic Compression

Liquefaction

'Liquefaction' refers to the loss of soil strength during a seismic event. The phenomenon is observed in areas that include geologically 'younger' soils (i.e., soils of Holocene age), shallow water table (less than about 60 feet depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. The seismic ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, which causes the soils to lose strength. Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

Neither the clayey fill, nor the very dense and geologically 'older' bedrock have potential for liquefaction.

Seismic Compression

Apart from liquefaction, a strong seismic event can induce settlement within loose to moderately dense, unsaturated granular soils. The clayey fill atop the bedrock will not be prone to seismic compression.

5.2.6 Expansion Potential

Expansive soils are clayey soil characterized by their ability to undergo significant volume changes (shrinking or swelling) due to variations in moisture content, the magnitude of which is related to both clay content and plasticity index. These volume changes can be damaging to structures. Nationally, the annual value of real estate damage caused by expansive soils is exceeded only by that caused by termites.

The clayey portions of the fill have been characterized by testing to determine Atterberg Limits (after ASTM D4318) and Expansion Index ('EI', after ASTM D4829).

Testing to determine Atterberg Limits shows that the clayey fill of the near subsurface beneath the prospective two-story building is of high plasticity, which is anticipated to have moderate to high expansion potential. The clayey fill in the near subsurface beneath the remainder of the site is of lower plasticity, with very low expansion potential ($EI = 7$).

5.2.7 Compressible Undocumented Fill

Review of historic aerial images of this site indicates that the existing groundform was created by relatively larger scale filling, that ranges up to approximately 35 feet in depth. The clayey fill in the near subsurface is relatively heterogeneous, ranging from high to low plasticity. The fill in the near subsurface is locally softer and will be compressible under loads from any new fills and/or the planned structures.

5.2.8 Corrosivity and Sulfate Attack

Chemical testing of the near-surface soils indicates the soils contain low concentrations of soluble sulfates and chlorides. The site soils will be mildly corrosive to unprotected embedded metals. There is no risk of sulfate attack to embedded concrete.

5.3. Siting Hazards

5.3.1 Effect on Adjacent Properties

As currently envisioned the proposed church will not affect the structural integrity of adjacent properties, existing public improvements, or street rights-of-way located adjacent to the site.

5.3.2 Flooding, Tsunamis, and Seiches

The project site is mapped within an area of minimal flood hazard (FEMA, 2012). The site is not located within a mapped area on the State of California Tsunami Inundation Maps (CGC, 2022a); therefore, damage due to tsunamis is considered negligible. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The closest body of water is Lake San Marcos, located approximately 0.81 miles northeast; however, the potential for a seiche to affect the site is considered negligible.

5.3.3 Subsidence

The site is not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum). The potential for subsidence due to the extraction of fluids is considered very low.

6. CONCLUSIONS

The subsurface exploration identified two soil/rock units across the site.

1. Undocumented Fill (af). The existing site groundform was developed by extensive filling over a natural drainage channel. Undocumented artificial fill covers the site to depths of about 30 feet, averaging around 15 feet thickness in the area of new buildings. The fill is heterogeneous and characteristically finer grained (clayey sands to sandy clays). Locally, the clay is of high plasticity, with expansion potential that may range from medium to high. NOVA has no records regarding the placement and compaction of the fill; therefore, it is considered 'undocumented' and at risk for wide variations in quality and consistency. This fill is not considered suitable for support of the proposed development.
2. Mesozoic Metasedimentary and Metavolcanic Rock (Mzu). The undocumented fill overlies sound metamorphic rock.

Review of geologic, soil, and siting-related hazards common to this region of California, has identified two hazards with the potential to affect development of the church, as abstracted below.

1. Strong Ground Motion. The site is at risk for moderate-to-severe ground shaking in response to a large-magnitude earthquake ($M \sim 7$) during the lifetime of the planned structures. While strong ground motion could affect the site, there is no risk of liquefaction or related seismic phenomena. The seismic risk will be mitigated by seismic resistant design as required by the California Building Code.
2. Compressible Fill. The undocumented fill described above is locally softer and will be compressible under loads from any new fills and/or the planned structures.

The site of the proposed construction is feasible for development from a geotechnical standpoint. It is expected that both the church and the two buildings may be supported on ground improved by stone columns, as described in the following section. The ground level slabs for each of these structures may be supported at grade after remedial grading per recommendations in the following section.

The granular and lower plasticity portions of the existing fill may be reused to create the new engineered fill; however, high plasticity clay is not suitable for reuse as engineered fill and should be disposed of in landscape areas or hauled off-site. Additionally, oversized material is anticipated to be encountered, which will have to be screened and properly disposed of.

Though no percolation testing was undertaken to address stormwater infiltration, it is expected that by virtue of the thick fills, this site would be a 'no infiltration' condition.

Groundwater was encountered in Borings B-2 and B-4, but if stone columns are used for construction, this water will not be a constraint to development.

When building locations and grading plans are finalized, NOVA should provide an updated report specific to the proposed development.

7. EARTHWORK, FOUNDATIONS, PAVEMENTS, AND STORMWATER

7.1. General

This section is intended to provide guidance for the geotechnical requirements associated with development of structures and infrastructure.

As is noted in Section 1, this report is limited only to a geotechnical-related assessment of the subject property, undertaken for its stated, limited objective only. Future work to develop or otherwise modify the property will require more in-depth geotechnical investigation. The recommendations of this section should be understood to be preliminary, subject to change as more site development and subsurface information is obtained.

In particular, the site is underlain by relatively thick undocumented fill. Borings conducted for this assessment showed the fill to be relatively heterogeneous. The design basis geotechnical assessment of this site should be relatively more intensive than is common for projects of this scale to address this heterogeneity.

7.2. Earthwork

7.2.1 General

Earthwork should be performed in accordance with Section 300 of the most recent approved edition of the *“Standard Specifications for Public Works Construction”* and *“Regional Supplement Amendments.”*

7.2.2 Potential Effects of Past Site Use

As is discussed in Section 2, the first use of the site was the mass grading completed in about 1994. Thereafter, the site was used as a nursery. Based upon these uses and the indications of the boring logs, there should be no relic subsurface structures (e.g., piping, foundations, etc.) that could affect foundation and utility construction planned for the office building. It is possible that some of the fill may include construction debris, contamination by regulated constituents (herbicides, or pesticides, for example), or unsuitable levels of organics.

7.2.3 Excavation Characteristics

It is anticipated that excavations can be achieved with conventional earthwork equipment in good working order. However, very dense formational materials occur in the near subsurface at the southwest of the site. Removals in this area may require extra excavation effort.

7.2.4 Select Fill

Material Requirements

Any fill or backfill for structures should be ‘Select Fill’, a mineral soil free of organics and regulated constituents with the characteristics listed on the following page:

- with at least 40% by weight finer than ¼ inches in size;
- maximum particle size of 6 inches;
- classified as CL, SW, SM, SC, GW, GM after ASTM D2487; and
- expansion index (EI) less than 20 (i.e., $EI < 20$, after ASTM D4829).

Some of the fill now in place will conform to the above criteria or can be processed to conformance. Some of the high plasticity clay, and oversized material may have to be hauled off the site.

Placement Requirements

Prior to placement, the upper 12 inches of the ground to receive Select Fill should be densified/compacted to a minimum of 90% relative compaction after ASTM D1557 (the 'modified Proctor').

Following moisture conditioning to at least 2% above the optimum moisture content, Select Fill should be placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift to at least 90% relative compaction after ASTM D1557. For most self-propelled construction equipment, this will limit loose lifts to about 8 inches or less, applying a minimum of four passes in the forward direction. Lift thickness for hand-operated equipment (tamper, walk-behind compactors, etc.) will be limited to on the order of 3 inches or less.

7.2.5 Remedial Grading - Building Pads

Beneath the proposed building slab on grade, existing fill soils should be excavated to 3 feet below planned finished grade. Horizontally, excavations should extend at least 3 feet outside the planned slab. The excavated soil should be replaced to finished pad grade with Select Fill in conformance with Select Fill per Section 7.2.4.

7.2.6 Remedial Grading - Pavements

Beneath proposed pavement areas, the existing soils should be excavated to a depth of at least 3 feet below existing grade or planned subgrade elevation, whichever is deeper. Horizontally, excavations should extend at least 2 feet outside the planned pavement or up to existing improvements or the limits of grading, whichever is less. The excavated soil should be replaced to finished pad grade with Select Fill in conformance with Section 7.2.4. The upper 12 inches of subgrade should be compacted to at least 95% relative compaction. If competent formational rock is exposed, excavation need not be performed.

7.3. Shallow Foundations with Ground Improvement

7.3.1 General

The requirements for deep remedial grading of the existing undocumented fill can be eliminated (with the exception of remedial grading below building slabs described above) by the use of ground improvement to provide support for shallow foundations.

Ground improvement of the clayey fill can be used to control total settlement of building foundations to about 0.5 inch with acceptable angular distortion. A variety of alternatives are available to this end, as are summarized in Figure 7-1. In consideration of the clayey fill and the relatively light structural loads, NOVA believes that aggregate piers might be the most cost-effective means to improve the clayey fill beneath foundations.

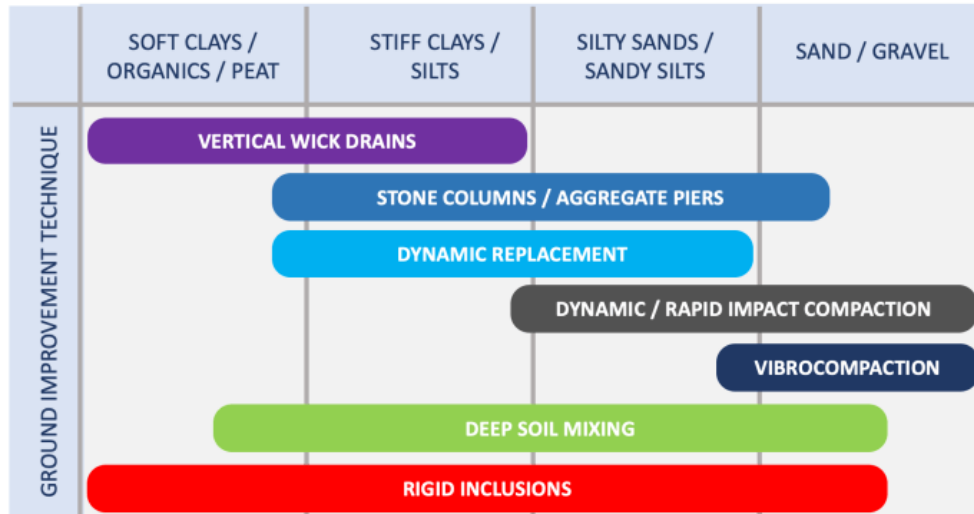


Figure 7-1. Ground Improvement Alternatives for Soils of Varying Gradation
(source: Civil + Structural Engineer, March 2021 (found at: <https://csengineermag.com>))

Aggregate piers are columns of compacted stone installed in groups to increase soil bearing pressure and mitigate settlement under structural footings. Within the industry, the term ‘aggregate pier’ may be used to describe a stiff stone column constructed by either:

- mechanical ramming (creating a ‘rammed aggregate pier’ (RAP), or
- high-frequency, high-energy vibration, creating a vibrated pier (also called a stone column).

Stone columns and RAPs are widely used in the region, often as a cost-effective means for increasing the stiffness of a softer soil mass. Ground improvement technologies, including aggregate piers, are vended by a variety of specialty contractors in the region. Design is normally completed by the specialty contractor and reviewed by the Geotechnical Engineer-of-Record (GEOR). As employed at this site, the principal parameters for design would be approximately as described on the following page.

7.3.2 Stone Columns

Stone columns are relatively quickly installed. Columns would be designed to extend from just below the footings to the formational rocks. It is expected that if stone columns are employed below foundations for all three buildings at this site, these columns would provide allowable bearing pressures on the order of 4,000 psf or greater. NOVA estimates installation could be completed over a period of about three weeks.

7.4. Preliminary Pavement Sections

An R-value of 10 was assumed for the design of preliminary pavement sections. The actual R-value of the subgrade soils should be determined after grading, and the final pavement sections should be provided. Based on an R-value of 10, the following preliminary pavement structural sections are provided for the assumed Traffic Indexes in Table 7-1.

Table 7-1. AC Pavement Sections

Traffic Type	Traffic Index	Asphalt Concrete (inches)
Parking Stalls	4.5	3 AC / 8 AB
Driveways	6.0	4 AC / 11 AB
Fire Lanes	7.5	3 AC / 13 AB

Note: AC: Asphalt Concrete AB: Aggregate Base

Subgrade preparation per Section 7.2 should be performed immediately prior to placement of the pavement section.

7.5. Stormwater

Though no percolation testing was undertaken to address stormwater infiltration, it is expected that by virtue of the thick fills, this site would be a 'no infiltration' condition.

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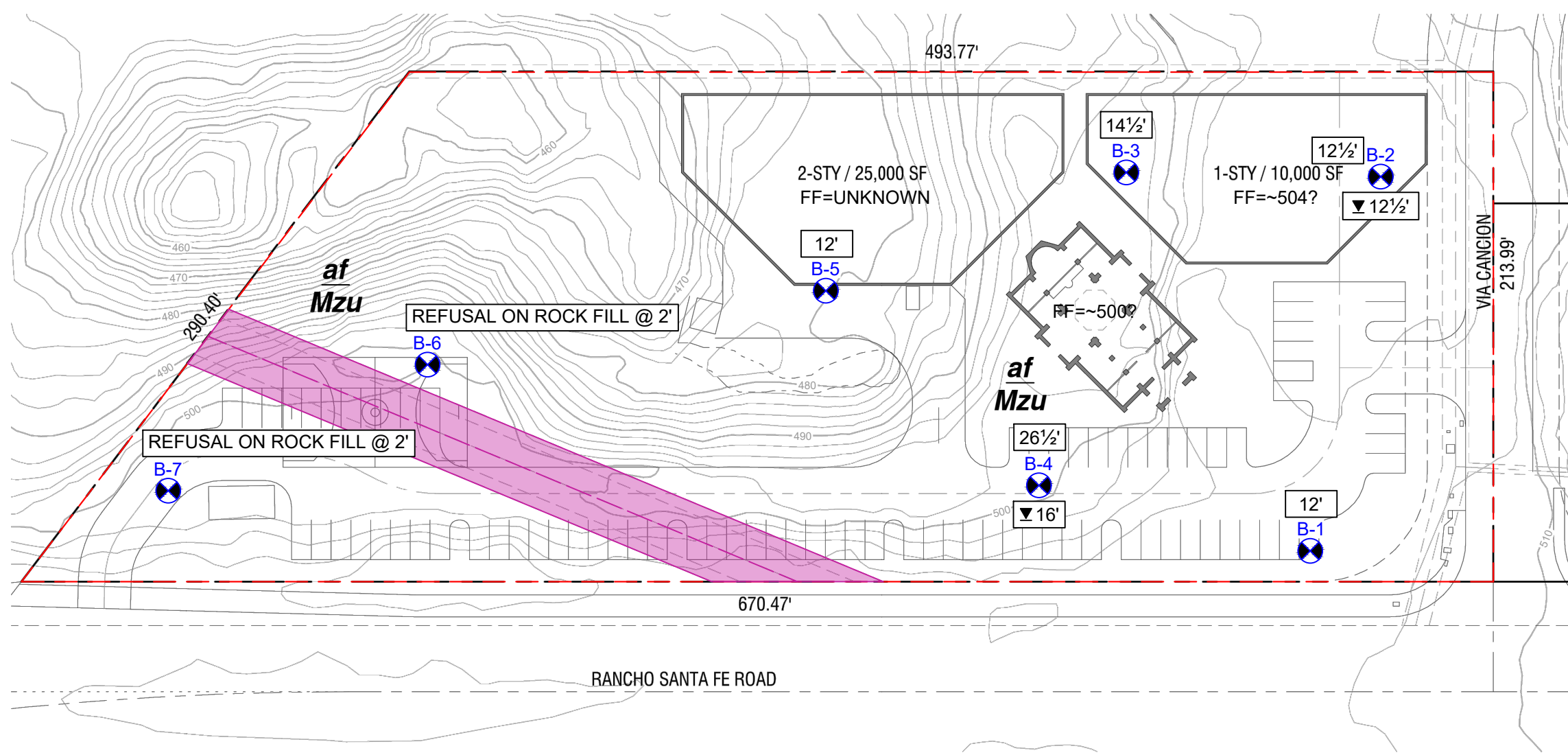


July 21, 2023

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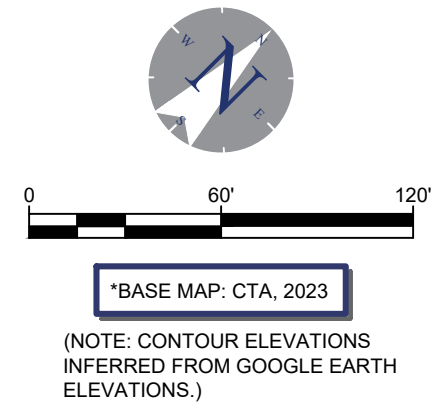


PLATES



KEY TO SYMBOLS

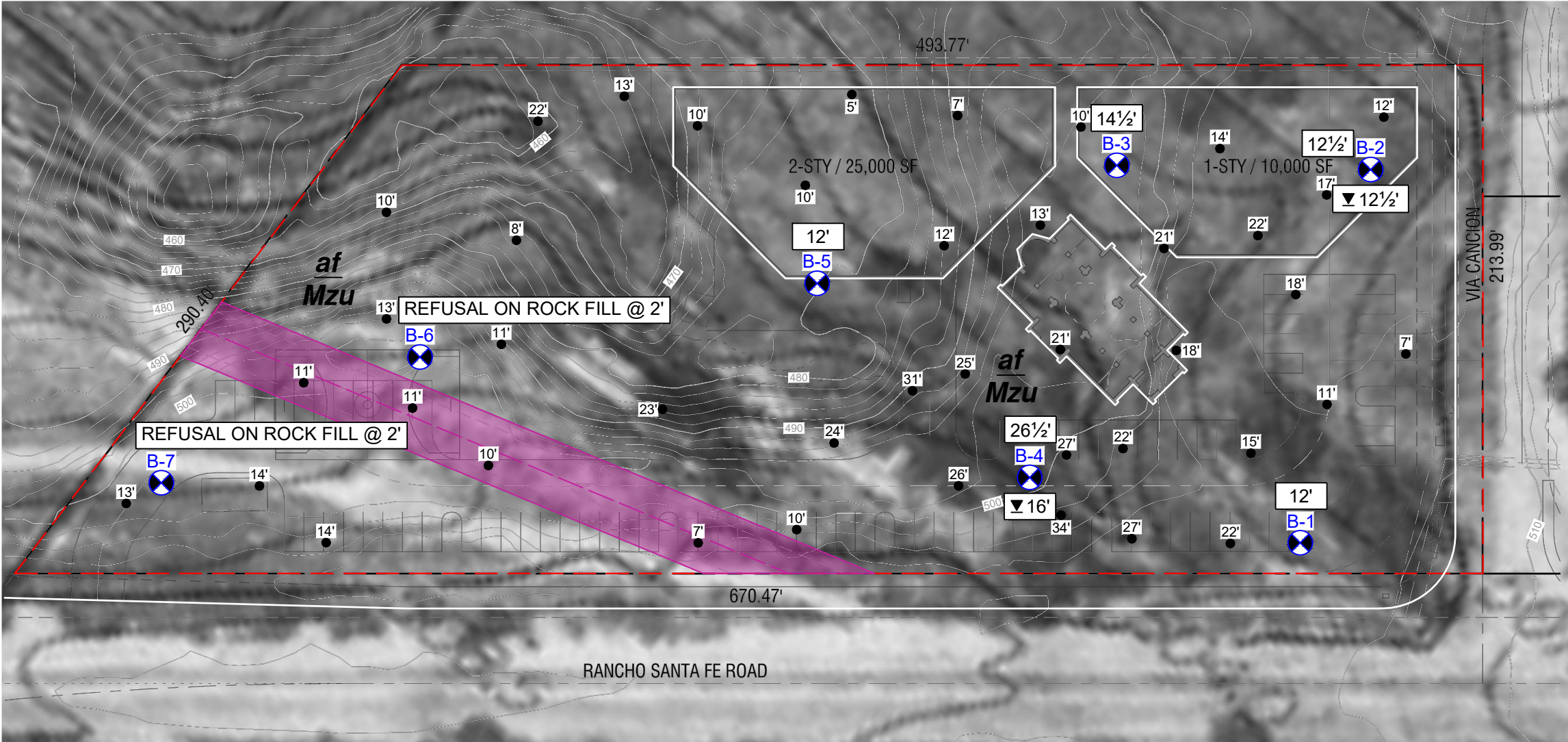
- af** FILL
- Mzu** MESOZOIC METASEDIMENTARY AND METAVOLCANIC ROCK
-  GEOTECHNICAL BORING
-  FILL DEPTH
-  GROUND WATER DEPTH (WHERE ENCOUNTERED)
-  LIMITS OF INVESTIGATION
-  PRESSURIZED SEWER MAIN
-  SEWER EASEMENT



**ST. KATHERINE OF ALEXANDRIA
ORTHODOX CHURCH**
1690 VIA CANCION
SAN MARCOS, CA 92078

PROJECT NO.:	2023084
DRAWN BY:	DJ
REVIEWED BY:	MS
SCALE:	1"=60'
DRAWING TITLE:	

GEOTECHNICAL MAP



KEY TO SYMBOLS

- af** FILL
- Mzu** MESOZOIC METASEDIMENTARY AND METAVOLCANIC ROCK
- B-7** GEOTECHNICAL BORING
- 31'** APPROXIMATE DEPTH OF FILL BASED ON HISTORICAL TOPO
- 26 1/2'** FILL DEPTH
- ▼ 16'** GROUND WATER DEPTH (WHERE ENCOUNTERED)
- LIMITS OF INVESTIGATION
- PRESSURIZED SEWER MAIN
- SEWER EASEMENT



*BASE MAP: CTA, 2023
*TOPO MAP: SDCC, 1975

(NOTE: CONTOUR ELEVATIONS
INFERRED FROM GOOGLE EARTH
ELEVATIONS.)

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PROJECT NO.: 2023084
DRAWN BY: DJ
REVIEWED BY: MS
SCALE: 1"=60'
DRAWING TITLE:

HISTORIC
TOPOGRAPHIC MAP



APPENDIX A

USE OF THE GEOTECHNICAL REPORT

Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the report for any purpose or project except the one originally contemplated.*

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time to perform additional study.* Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that

have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant: ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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APPENDIX B

BORING LOGS

MAJOR DIVISIONS			TYPICAL NAMES	
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVEL MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVEL WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVEL WITH OR WITHOUT SAND
			GP	POORLY GRADED GRAVEL WITH OR WITHOUT SAND
		GRAVEL WITH 15% OR MORE FINES	GM	SILTY GRAVEL WITH OR WITHOUT SAND
			GC	CLAYEY GRAVEL WITH OR WITHOUT SAND
	SAND MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 15% FINES	SW	WELL-GRADED SAND WITH OR WITHOUT GRAVEL
			SP	POORLY GRADED SAND WITH OR WITHOUT GRAVEL
		SAND WITH 15% OR MORE FINES	SM	SILTY SAND WITH OR WITHOUT GRAVEL
			SC	CLAYEY SAND WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	SILT WITH OR WITHOUT SAND OR GRAVEL
			CL	LEAN CLAY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILT OR CLAY OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	ELASTIC SILT WITH OR WITHOUT SAND OR GRAVEL
			CH	FAT CLAY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILT OR CLAY OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

 GROUNDWATER / STABILIZED

 GROUNDWATER SEEPAGE

 BULK SAMPLE

 SPT SAMPLE (ASTM D1586)

 MOD. CAL. SAMPLE (ASTM D3550)

 UNRELIABLE BLOW COUNTS

 GEOLOGIC CONTACT

 SOIL TYPE CHANGE

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

LOG ABBREVIATIONS

REF REFUSAL

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	SPT N60 BLOWS/FOOT	CONSISTENCY	SPT N60 BLOWS/FOOT	POCKET PENETROMETER MEASUREMENT (TSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.24
LOOSE	5 - 10	SOFT	3 - 4	0.25 - 0.49
MEDIUM DENSE	11 - 30	MEDIUM STIFF	5 - 8	0.50 - 0.99
DENSE	31 - 50	STIFF	9 - 15	1.0 - 1.99
VERY DENSE	OVER 50	VERY STIFF	16 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

IF THE SEATING INTERVAL (1st 6 INCH INTERVAL) IS NOT ACHIEVED, N IS REPORTED AS REF. IF MORE THAN 50 BLOWS ARE RECORDED IN ANY 6-INCH INTERVAL, N₆₀ IS REPORTED AS REF.

LOG OF BORING B-1

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)

ELEVATION (FT): +507 MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB

GROUNDWATER DEPTH (FT): N/A NOTES: ETR~73.9%, N₆₀ ~ $\frac{73.9}{60} \times N \sim 1.23 \times N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N ₆₀	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							CH/SC	FILL: SANDY CLAY/CLAYEY SAND; GRAYISH BROWN, MOIST, SOFT TO MEDIUM STIFF/LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, MICACEOUS, SOME GRAVEL VERY MOIST	
5			4 6 6	15			SC	CLAYEY SAND; YELLOWISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE ROOTS AND GRAVEL AT 6½ FT AUGER CHATTER DUE TO GRAVEL IN THE FILL	
10			13 18 32	62*				POORLY MIXED YELLOWISH BROWN CLAYEY SAND WITH DARK BROWN SANDY CLAY THROUGHOUT, UNRELIABLE BLOW COUNTS DUE TO SAMPLER BOUNCING ON ROCK IN THE FILL, FINE TO COARSE GRAINED, ABUNDANT GRAVEL, TRACE CONCRETE FRAGMENTS	
			50/5"	REF				SIGNIFICANT AUGER CHATTER, NO SAMPLER RECOVERY	
15								BORING TERMINATED AT 13 FT DUE TO AUGER REFUSAL ON VERY DENSE METASEDIMENTARY AND METAVOLCANIC ROCK. NO GROUNDWATER ENCOUNTERED.	
20									
25									
30									



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ST. KATHERINE OF ALEXANDRIA ORTHODOX CHURCH

1690 VIA CANCION
SAN MARCOS, CA 92078

DRAFTED BY: DTB

PROJECT: 2023084

APPENDIX: B.1

LOG OF BORING B-2

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)
 ELEVATION (FT): +505 MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB
 GROUNDWATER DEPTH (FT): 11½ NOTES: ETR~73.9%, N₆₀ ~ $\frac{73.9}{60} \times N \sim 1.23 \times N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N ₆₀	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							CH	FILL: SANDY FAT CLAY; GRAYISH BROWN, MOIST, SOFT TO MEDIUM STIFF, FINE TO MEDIUM GRAINED SAND, ABUNDANT GRAVEL	
5			4 7 12	23			CH	POORLY MIXED SANDY CLAY, GRAYISH BROWN AND BLuish GRAY, MOIST, STIFF, FINE GRAINED SAND, TRACE ROOTS, POCKET PENETROMETER TEST = 1.5 TSF	
10			22 7 10	21			SC	DRILLER REPORTS INCREASED DRILLING RESISTANCE, AUGER CHATTER DUE TO GRAVEL IN THE FILL CLAYEY SAND; DARK GRAY, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, MICACEOUS, TRACE ROOTS, GRAVEL AT 10½ FT	
15			29 8 8	20				WET, PERCHED GROUNDWATER AT 11½ FT, ABUNDANT GRAVEL	
20			34 50/6"	REF				MESOZOIC METASEDIMENTARY AND METAVOLCANIC ROCK, UNDIVIDED (Mzu): SILTY SAND; GRAYISH BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED, MICACEOUS	
25			50/6"	REF				SILTY SANDSTONE; GRAYISH BROWN, MOIST, VERY DENSE, FINE TO MEDIUM GRAINED, MICACEOUS	
30								GRAYISH BROWN WITH REDDISH YELLOW MOTTLING BORING TERMINATED AT 19½ FT. PERCHED GROUNDWATER ENCOUNTERED AT 11½ FT.	



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1690 VIA CANCION
SAN MARCOS, CA 92078

DRAFTED BY: DTB

PROJECT: 2023084

APPENDIX: B.2

LOG OF BORING B-3

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)
 ELEVATION (FT): +498 MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB
 GROUNDWATER DEPTH (FT): N/A NOTES: ETR~73.9%, N₆₀ ~ $\frac{73.9}{60} \times N \sim 1.23 \times N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N ₆₀	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							CH	FILL: SANDY FAT CLAY; GRAYISH BROWN, MOIST, SOFT TO MEDIUM STIFF, FINE GRAINED SAND	SA AL EI
5			5 10 11	26			SC/CH	POORLY MIXED YELLOWISH BROWN CLAYEY SAND WITH DARK BROWN SANDY FAT CLAY THROUGHOUT, MEDIUM DENSE, FINE TO MEDIUM GRAINED SAND, TRACE GRAVEL	
10			6 13 8	26			CH	YELLOWISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED SANDY FAT CLAY; GRAY, MOIST, STIFF, FINE TO MEDIUM GRAINED SAND, POCKET PENETROMETER TEST = 1.0 TSF	
15			13 27 50/5"	REF				MESOZOIC METASEDIMENTARY AND METAVOLCANIC ROCK, UNDIVIDED (Mzu): CLAYEY SANDSTONE; YELLOWISH BROWN, GRAY WITH REDDISH YELLOW MOTTLING, MOIST, VERY DENSE, FINE TO MEDIUM GRAINED, DRILLER REPORTS AN INCREASE IN DRILLING RESISTANCE	
			50/4"	REF				BORING TERMINATED AT 16 FT DUE TO AUGER REFUSAL ON VERY DENSE METASEDIMENTARY AND METAVOLCANIC ROCK. NO GROUNDWATER ENCOUNTERED.	
20									
25									
30									



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ST. KATHERINE OF ALEXANDRIA ORTHODOX CHURCH

1690 VIA CANCION
SAN MARCOS, CA 92078

DRAFTED BY: DTB

PROJECT: 2023084

APPENDIX: B.3

LOG OF BORING B-4

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)

ELEVATION (FT): +500 MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB

GROUNDWATER DEPTH (FT): 16 NOTES: ETR~73.9%, $N_{60} \sim \frac{73.9}{60} N \sim 1.23^*N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							SC	FILL: POORLY MIXED CLAYEY SAND AND RECYCLED AGGREGATE BASE ROCK, YELLOWISH AND GRAYISH BROWN, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED AUGER CHATTER DUE TO GRAVEL IN THE FILL AUGER CHATTER STOPS, DRILLING RESISTANCE DECREASES	
5			6 6 7	16				MEDIUM DENSE, MICACEOUS AUGER CHATTER DUE TO GRAVEL IN THE FILL	
10			34 13 13	32			AC	ASPHALT CONCRETE PAVEMENT; DARK GRAY BROWN, DRY, DENSE	
							SC	CLAYEY SAND; GRAYISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED	
15			15 28 50/4"	62*				PARTIAL RECOVERY, WET, UNRELIABLE BLOW COUNTS, SAMPLER BOUNCING ON ROCK GROUNDWATER AT 16 FT	
20			4 6 7	16			ML	SANDY SILT; YELLOWISH BROWN, WET, MEDIUM DENSE/VERY STIFF, VERY FINE GRAINED SAND	
25							CH/CL	SANDY CLAY; DARK GRAY, WET, SOFT, FINE GRAINED SAND, TRACE ROOTS, SOME GRAVEL	
			50/2" 50/1"	REF REF				MESOZOIC METASEDIMENTARY AND METAVOLCANIC ROCK, UNDIVIDED (Mzu): SANDY SILTSTONE/SILTY SANDSTONE; YELLOWISH BROWN WITH GRAY MOTTLING, WET, VERY DENSE/HARD, VERY FINE GRAINED SAND, TRACE GRAVEL, SIGNIFICANT AUGER CHATTER	
30								BORING TERMINATED AT 27 FT. GROUNDWATER ENCOUNTERED AT 16 FT.	



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1690 VIA CANCION
SAN MARCOS, CA 92078

DRAFTED BY: DTB

PROJECT: 2023084

APPENDIX: B.4

LOG OF BORING B-5

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)
 ELEVATION (FT): +479½ MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB
 GROUNDWATER DEPTH (FT): N/A NOTES: ETR~73.9%, N₆₀ ~ $\frac{73.9}{60} \times N \sim 1.23 \times N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N ₆₀	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							SC/GP	FILL: POORLY MIXED CLAYEY SAND AND RECYCLED AGGREGATE BASE ROCK, YELLOWISH AND GRAYISH BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, ABOUT 11 - 25% GRAVEL	SA AL EI CR
5								<i>AUGER CHATTER DUE TO GRAVEL IN THE FILL</i>	
			12 7 8	19			AC SC	ASPHALT CONCRETE PAVEMENT; DARK GRAY BROWN, DRY, MEDIUM DENSE CLAYEY SAND; GRAYISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED	
10			9 8 12 11	25			CH/CL	SANDY CLAY; DARK BROWN, MOIST, VERY STIFF, FINE GRAINED SAND, POCKET PENETROMETER TEST = 2.75 TSF, REDDISH YELLOW AT 11½ FT	
			16 25 50/6"	50 REF				MESOZOIC METASEDIMENTARY AND METAVOLCANIC ROCK, UNDIVIDED (Mzu): CLAYEY SANDSTONE; REDDISH YELLOW WITH BROWN MOTTLING, MOIST, VERY DENSE, FINE TO MEDIUM GRAINED, PARTIAL RECOVERY, DRILLER REPORTS SAMPLER BOUNCING ON ROCK	
15								BORING TERMINATED AT 14 FT DUE TO AUGER REFUSAL ON VERY DENSE METASEDIMENTARY AND METAVOLCANIC ROCK. NO GROUNDWATER ENCOUNTERED.	
20									
25									
30									



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APPENDIX: B.5

LOG OF BORING B-6

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)
 ELEVATION (FT): +500 MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB
 GROUNDWATER DEPTH (FT): N/A NOTES: ETR~73.9%, N₆₀ ~ $\frac{73.9}{60} \times N \sim 1.23 \times N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N ₆₀	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							SC	FILL: CLAYEY SAND; YELLOWISH BROWN, LOOSE, FINE GRAINED, SOME GRAVEL	
5								BORING TERMINATED AT 2 FT DUE TO SHALLOW REFUSAL ON LARGE DIAMETER BOULDERS. RELOCATION WAS NOT POSSIBLE DUE TO THE 20-FT EASEMENT FOR THE PRESSURIZED SEWER MAIN, PER THE VALLECITOS WATER DISTRICT. NO GROUNDWATER ENCOUNTERED.	
10									
15									
20									
25									
30									



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PROJECT: 2023084

APPENDIX: B.6

LOG OF BORING B-7

DATE DRILLED: JUNE 8, 2023 DRILLING EQUIPMENT: CME 75 SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTO)
 ELEVATION (FT): +500½ MSL DRILLING METHOD: 8-INCH HOLLOW STEM AUGER LOGGED BY: DTB
 GROUNDWATER DEPTH (FT): N/A NOTES: ETR~73.9%, N₆₀ ~ $\frac{73.9}{60} \times N \sim 1.23 \times N$ REVIEWED BY: MS

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER 6 IN N	N ₆₀	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0	X						SC	FILL: CLAYEY SAND; YELLOWISH BROWN, LOOSE, FINE GRAINED, ABOUT 11 - 25% GRAVEL	SA AL
5								BORING TERMINATED AT 2 FT DUE TO SHALLOW REFUSAL ON LARGE DIAMETER BOULDERS. SIX RELOCATION ATTEMPTS WERE PERFORMED, 5 FT AWAY, EACH CONTAINED A SUBSURFACE OBSTRUCTION. NO GROUNDWATER ENCOUNTERED.	
10									
15									
20									
25									
30									



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APPENDIX: B.7



APPENDIX C

LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:

- **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soils Classification System and are presented on the exploration logs in Appendix B.
- **GRADATION ANALYSIS (ASTM D6913):** Gradation analyses were performed on representative soil samples in general accordance with ASTM D422. The grain size distributions of the samples were determined in accordance with ASTM D6913.
- **ATTERBERG LIMITS (ASTM D4318):** Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limits, plastic limits, and plasticity indexes in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System.
- **EXPANSION INDEX (ASTM D4829):** The expansion index of selected material was evaluated in general accordance with ASTM D4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and were inundated with distilled water. Readings of volumetric swell were made for a period of 24 hours.
- **CORROSIVITY TEST (CAL. TEST METHOD 417, 422, 643):** Soil pH and minimum resistivity tests were performed on a representative soil sample in general accordance with test method CT 643. The sulfate and chloride contents of the selected sample was evaluated in general accordance with CT 417 and CT 422, respectively.

Soil samples not tested are now stored in our laboratory for future reference and evaluation, if needed. Unless notified to the contrary, samples will be disposed of 60 days from the date of this report.



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LAB TEST SUMMARY

ST. KATHERINE OF ALEXANDRIA ORTHODOX CHURCH

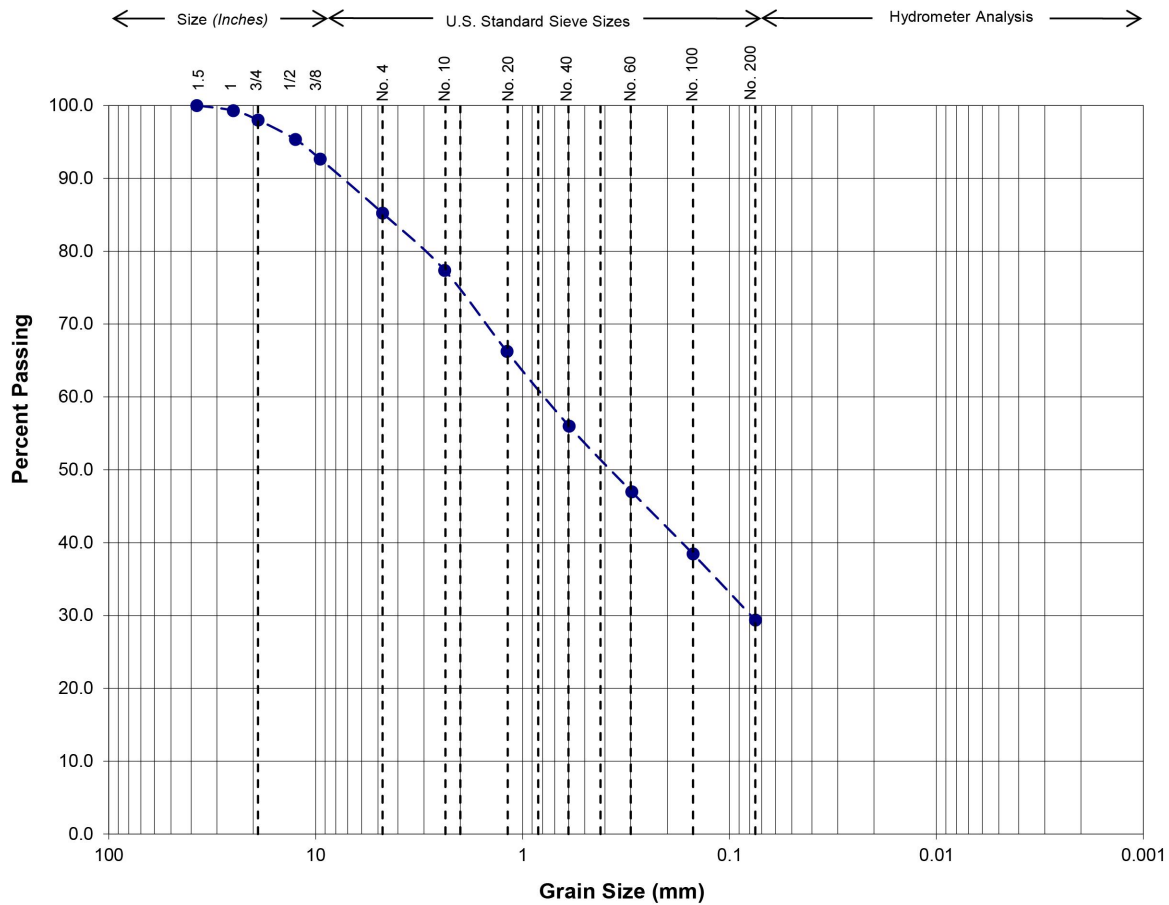
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FIGURE: C.1



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B-5	Atterberg Limits (ASTM D4318):	
Depth (ft):	0 - 5	Liquid Limit, LL:	35
USCS Soil Type:	SC	Plastic Limit, PL:	13
Passing No. 200 (%):	29	Plasticity Index, PI:	22



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CLASSIFICATION TEST RESULTS

ST. KATHERINE OF ALEXANDRIA ORTHODOX CHURCH

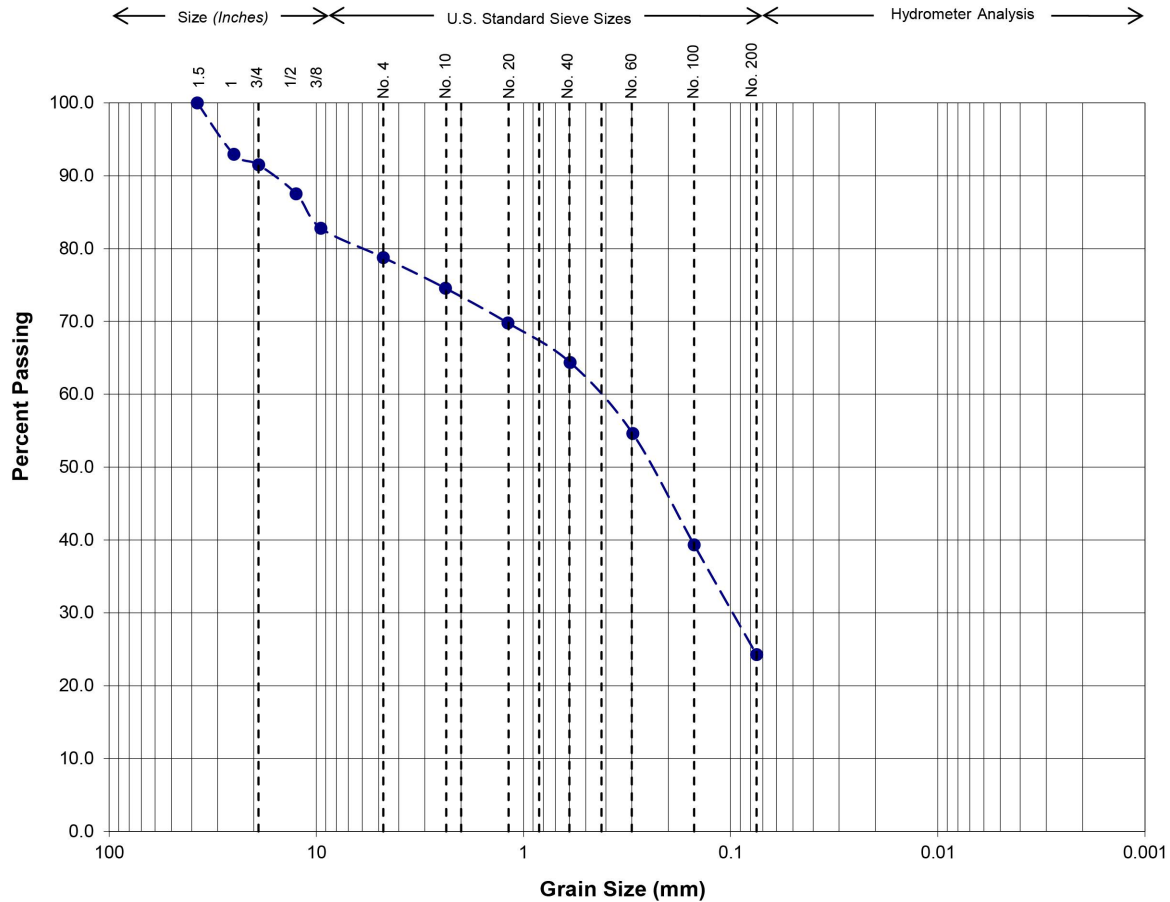
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FIGURE: C.2



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B-7	<u>Atterberg Limits (ASTM D4318):</u>	
Depth (ft):	0 - 2	Liquid Limit, LL:	29
USCS Soil Type:	SC	Plastic Limit, PL:	13
Passing No. 200 (%):	24	Plasticity Index, PI:	16



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CLASSIFICATION TEST RESULTS

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FIGURE: C.3

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft.)	Expansion Index	Expansion Potential
B-5	0 - 5	7	Very Low

Classification of Expansive Soil (ASTM D4829)

Expansion Index	Expansion Potential
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

Corrosivity (Cal. Test Method 417,422,643)

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
B-5	0 - 5	8.12	1706	39	0.004	117	0.012

Water-Soluble Sulfate Exposure (ACI 318 Table 19.3.1.1 and Table 19.3.2.1)

Water-Soluble Sulfate (SO ₄) in Soil (% by Weight)	Exposure Severity	Exposure Class	Cement Type (ASTM C150)	Max. W/C	Min. f _c ' (psi)
SO ₄ < 0.10	N/A	S0	No type restriction	N/A	2,500
0.10 ≤ SO ₄ < 0.20	Moderate	S1	II	0.50	4,000
0.20 ≤ SO ₄ ≤ 0.20	Severe	S2	V	0.45	4,500
SO ₄ > 2.00	Very Severe	S3	V plus pozzolan or slag cement	0.45	4,500



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LAB TEST RESULTS

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FIGURE: C.4